D R A F T   R E P O R T

IN-DELTA STORAGE PROGRAM EMBANKMENT DESIGN ANALYSIS

Prepared for
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1.1 PURPOSE

The Department of Water Resources (DWR) is conducting feasibility-level engineering and environmental studies under the Integrated Storage Investigations Program. As part of the project evaluations, DWR is evaluating the technical feasibility and conducting engineering investigations for the In-Delta Storage Program. Engineering investigation will aim at developing solutions to enhance project reliability through improved embankment design and consolidation of inlet and outlet structures.

As part of this feasibility study, the Department requests that URS Corporation (URS) undertake a detailed risk analysis and integrate the physical design with a desirable level of protection through seismic, flooding, operational, environmental and economic analyses. Other objectives are to recommend a desirable level of protection and an appropriate factor of safety for the project.

1.2 SCOPE OF WORK

This report presents the evaluation of the vulnerability and reliability of the existing conditions and In-Delta Storage Re-engineered project (embankment and integrated facilities) under Operational Demands. The specific tasks proposed under statement of work for the Task Order No. IDS-0702-1747-002 are presented below.

Task 1 – Collect and Review Existing Information

Update information on aquifer, groundwater, and soil data. Data includes monitoring well data, tidal gage readings in the Delta, previous aquifer tests, and recent (proposed) field and laboratory test results. Review reservoir operation criteria and proposed stage curve. Review recent relevant publications related to the delta levee seepage and stability conditions. Update geometry and levee cross-section information for the existing conditions, if additional topographic surveys are available. Evaluate strengthening or modifications to the existing levees cross-sections for the re-engineered embankment.

Task 2 – Develop Material Properties and Establish Analysis Criteria

Develop confidence levels around the best estimate profile, material, and stratigraphic information to assess the uncertainty and potential variations of these parameters for the existing conditions. Establish material properties and analysis criteria based on the recent planning study entitled “In-Delta Storage Program, Draft Report on Engineering Investigations,” dated May 2002 and obtain consensus for use in the following analyses.

Task 3 – Perform Stability and Seepage Analyses

Estimate seepage conditions (using the computer program SEEP-W) and conduct static stability analyses (using the limit equilibrium code UTEXAS3) for the existing levee conditions and for the proposed re-engineered project. The seepage analyses will include two representative cross-sections for each island. The analysis for the existing conditions will include high water level in the sloughs and empty island interiors. This analysis will be used as a baseline as well as to evaluate the risk of failure of the existing islands. The seepage analyses for the re-engineered project will include three cases consisting of: 1) completed embankment with reservoir empty and high water level in the sloughs; 2) full reservoir with the interceptor well not in operation;

and 3) full reservoir with interceptor wells in operation. The result of the last case will be linked to an aquifer model to estimate pressure heads and hydraulic gradients around the wells to estimate the point well effects.

The stability analyses will be conducted for the existing levees and the re-engineered embankment. For the existing levees the analysis will consider two cases consisting of the long-term steady-state condition under high water level in the delta and the minimum water level on the slough side. For the re-engineered embankment, the analysis will consider the following cases: immediately after construction, long-term steady state, and rapid drawdown for four representative cross-sections. For the long-term condition, the geometry of the embankment will be adjusted to account for the effect of consolidation settlement and subsidence. Yield accelerations will be calculated for the long-term conditions for the most critical slip surfaces for each cross-section. These values will be used with the seismic risk Task Order.

Task 4 – Estimate Probability of Failure

Estimate the probability of failure of the identified failure modes under operational conditions. These include: 1) internal erosion and piping due to high exit gradient caused by excessive seepage, 2) erosion through cracks in the levees (existing) and embankment (re-engineered) caused by differential settlement or unstable slopes, and 3) overtopping caused by slumping or loss of freeboard due to slope failure or excessive settlement. Calculate the probabilities of failure for all potential triggering events and failure modes. Calculate the aggregated probability of failure for triggering events and failure modes for operational conditions. The operational risk will include the flood event up to the 300-year flood. The seismic events will not be considered. The flood risk, presented in a separate Task Order, addresses only the probability of overtopping as a result of inadequate freeboard.

2.1 DATA REVIEW

Several geotechnical and environmental studies have been conducted at the two proposed reservoir sites and neighboring islands. Reports conducted in the neighboring islands that were reviewed in this current study include: (1) Converse Ward Davis Dixon, Inc. (CCWD) (1981); (2) U. S. Army Corps of Engineers (USACE) (1982); (3) Roger Foott Associates, Inc. (RFA) (1991a, b, and c); and (4) RFA (1994). The more relevant studies conducted at Bacon Island and Webb Tract include: (1) A preliminary geotechnical investigation by Harding Lawson Associates (HLA) (1989); (2) An adjunct draft geotechnical report prepared by URS (2000, 2001); and (3) U. S. Bureau of Reclamation’s (USBR) Status Report for the Delta Wetlands Project (2001). DWR’s recent planning study entitled “In-Delta Storage Program, Draft Report on Engineering Investigations,” dated May 2002 was also reviewed.

In addition, we reviewed and incorporated the following data provided by DWR into the current study: (1) stick log profiles summarizing SPT borings conducted in 1958 and CPT soundings obtained in 2001; (2) strength of soft organic soils at Webb Tract and Bacon Island; and (3) recommended elevations for benches in the new embankments on the slough side.

We reviewed the geotechnical data, assumptions made and results contained in the above reports. These reports describe subsurface soil conditions encountered during various field and laboratory investigations. Previous field investigations included drilling and standard penetration testing (SPT), sampling, and cone penetration testing (CPT). Previous laboratory testing programs included engineering property determination of embankment material and foundation soil.

New field or laboratory work for the current study included a USBR exploration program consisting of 19 CPT soundings at Webb Tract and 18 CPT soundings at Bacon Island drilled in 2002. No other field or laboratory work was performed for this study.

2.2 ANALYSIS PARAMETERS

2.2.1 Subsurface Conditions

Longitudinal profiles of the subsurface conditions along the perimeter of both islands developed and described in URS (2001) were updated to include the CPT data obtained by USBR in 2002. In addition, these profiles were compared with stick-log profiles provided by DWR. No significant changes from previous interpretations of the stratigraphy under the levees was observed.

The general stratigraphy of the levee and underlying soils of Bacon Island and Webb Tract are similar. The stratigraphy of the interior of the islands consists of a surficial soft, organic fibrous peat (PT) layer underlain by a silty sand (SM) aquifer, below which lies stiff lean clay (CL). These units are laterally continuous and vary in thickness from one part of the island to another. The silty sand layer is exposed in some portions of Webb Tract. Deeper sand aquifers are present below the stiff clay in some areas.

The levees are typically built of about 10 feet of sandy to clayey fill, placed on a mixture of clayey peat and peat fill that overlies the natural peat layer. The levee fill consists of inter-fingered layers of sand, peat, clay and clayey peat, that is likely to have more sand on the land side and peat and peaty clay on the slough side. Portions of the sandy reservoir side of the levee fill may be loose based on the methods of placement used during construction of the levees. The underlying peat is fibrous, soft, and highly compressible. Based on the available data it is not feasible to differentiate the clayey peat and peat fill from the natural peat. The available data also suggests that the engineering properties of the materials are similar. Occasionally, up to 15 feet of fat organic clay (OH) are encountered between the peat and underlying silty sand layer. For this study, the clayey peat and peat fill, natural peat, and fat clay have been combined to make up one layer. The combined layer thickness ranges from 15 to 40 feet under the levees.

At Bacon Island, borings available for review indicate that the sand underlying the peat ranges from medium dense to very dense. However, liquefaction potential figures included in USACE (1987) indicate that the upper 2 to 13 feet of the silty sand layer underlying the peat under portions of the perimeter is loose and, therefore, potentially liquefiable. The sand underlying the loose sand typically ranges from medium dense to very dense.

At Webb Tract, borings available for review indicate that the upper 3 to 7 feet of the sand layer under portions of the perimeter is loose and, therefore potentially liquefiable. The interpretation of such condition was based on low uncorrected SPT blow counts (4 to 7) and/or simultaneous occurrence of low tip resistance and low friction ratio in the CPT logs. In addition, liquefaction potential figures included in USACE (1987) indicate the upper 2 to 16 feet of the underlying sand is potentially liquefiable. The sand underlying the loose sand ranges from medium dense to very dense.

For the current study, the upper five feet of the sand layer is assumed to be potentially liquefiable under portions of the perimeters of both islands. This thickness was based on the borings and CPT soundings available for our review. SPT borings from which liquefaction potential figures in USACE (1987) were determined were not reviewed.

The islands were divided into sections based on the elevation of the base of peat. The subdivision of the islands is shown in Tables 2-1 and 2-2. Bacon Island has been divided into four sections with the base of peat elevation ranging from –20 feet to –40 feet. Webb Tract has also been divided into four sections with the base of peat elevation ranging from –25 feet to –40 feet.

The subsurface conditions at Webb Tract Section 4, shown in Table 2-2, differ from those encountered elsewhere around the island. At that section, below the levee fill, approximately 40 feet of materials with relatively low CPT tip penetration resistance (averaging 50 tsf) and low friction ratio were encountered. Section 4 corresponds to a repaired portion of the levee. This localized section was not considered in this study.

Previous evaluations have shown that peat thickness under the levees has the greatest influence on slope instability. For the current study, two cases representing the new embankment constructed over peat having the highest (smallest peat thickness) and lowest (largest peat thickness) base elevations were analyzed. The cases are considered to be representative of both islands due to the similarity of the stratigraphy of the islands. The cases are as follows:

1. Peat at El. –20 feet with the bottom of levee fill at 0 feet
2. Peat at El. –40 feet with the bottom of levee fill at 0 feet

2.2.2 Embankment Geometry

The existing levees will be raised and strengthened, generally on the island side, to form the embankments impounding the proposed reservoirs. The configuration for the new embankments around both islands has a crest elevation of +10 feet, with a final crest width of 35 feet. The inside slope of the reservoir is a composite slope. The slope above elevation +4 feet is 3H:1V and the lower slope is 10H:1V. Erosion protection covers the inside slope from elevation +3 to the crest. Two configurations were considered for the slough-side slope. These are referred to in this study as the “rock berm” option and the “bench” option and are described in the following paragraphs.

2.2.2.1 “Rock Berm” Option

The “rock berm” option consists of constructing the new embankment on top of the existing levee as shown on Figure 2-1. The slough-side slope of the new embankment extends from the outboard crest of the existing levee toward the slough at a 3H:1V slope. Where the existing slough-side slope is steeper than 3H:1V, rock fill would be placed from the outboard crest of the existing levee outward to the bottom of the slough at a 3H:1V slope. Rockfill would also be placed from the outboard crest of the existing levee to the bottom of the slough at slopes flatter than 3H:1V where required to meet stability criterion. Free-draining reservoir side berms would be placed at the bottom of the reservoir-side slope toe where analyses of combinations of base of peat elevation and reservoir base elevation result in factors of safety that do not meet project criteria.

2.2.2.2 “Bench” Option

The “bench” option, shown on Figure 2-2, consists of a bench, created by removing a portion of the existing levee to an elevation varying between 0 and 6 feet and constructing the new embankment from the reservoir side of the bench at a slope of 3H:1V to the crest of the embankment. In addition to removing load from the slough side of the embankment in order to provide a stable slough-side slope, the bench provides opportunity for environmental mitigation. The bench shifts the crest of the new embankment towards the reservoir. Erosion protection covering the slough-side slope above the bench would consist of riprap and bedding. Free-draining reservoir side berms would be placed at the bottom of the reservoir-side slope toe where analyses of combinations of base of peat elevation and reservoir base elevation result in factors of safety that do not meet project criteria.

2.2.2.3 Existing Levee Geometry

The geometry of the existing levees around Bacon Island and Webb Tract vary with respect to reservoir side angle, crest width, crest elevation, slough side angle, and slough bottom. Geometric variations were developed for the sections shown in Tables 2-1 and 2-2. Plan views of Bacon Island and Webb Tract with stationing around the islands are shown in Figures 2-3 and 2-4. The geometry of the existing levee slopes, slough bottom, and reservoir bottom for the current study are summarized in Tables 2-3 and 2-4.

Based on conversations with DWR, and also on Hultgren Tillis (2002) letter report, rockfill exists on the slough-side slopes. Hultgren Tillis (2002) states that there has been loss of rock from the slopes (steeper than 2H:1V) from the Delta island levees. In addition, the extent and thickness of the rockfill is not known for certain. Therefore, the rockfill was not considered to be a continuous layer everywhere on the slough-side slopes and, as such, was not included in stability analyses.

2.2.2.3 Effect of Settlement

Construction of the new embankments over highly compressible organic soils in the foundation will result in significant settlement. Progressive placement of fill will be required to construct and maintain the final crest elevation resulting in substantial reduction of peat thickness under the embankments. The geometry of the new embankment fill and underlying peat for long term steady state stability conditions should incorporate the deformation due to consolidation of the peat.

The finite element code program, Plaxis version 7.0 [Brinkgreve and Vermeer (1998)], was used to estimate the deformed geometry at the end of consolidation. Plaxis is a finite element program specifically intended for the analysis of deformation and stability in geotechnical engineering problems. It provides advanced constitutive models for the simulation of the non-linear and time-dependent behavior of soils.

Compressibility data for peat reported in previous investigations was reviewed. The data indicate a variation of values for compressibility parameters with respect to water content of the peat. Due to the variation, compressibility parameters used for the study were based primarily on observed consolidation reported in previous investigations and a model constructed to duplicate rates of settlement observed during a levee test fill on similar soils. A summary of the compressibility parameters are shown in Table 2-5.

Deformed geometries were generated by constructing the new embankments in Plaxis using two or more construction stages until the desired geometry was achieved at the end of consolidation. The deformed geometries of the new embankment for Case 2 (base of peat at –40 feet) for the “rock berm” and “bench” options are shown on Figures 2-1 and 2-2. Deformed geometries for Case 1 (base of peat at –20 feet) are shown on the stability analysis figures included in Appendix A.

2.2.3 Material Properties

2.2.3.1 Stress-Strain-Strength Properties

Material properties were based on the recent planning study entitled “In-Delta Storage Program, Draft Report on Engineering Investigations,” dated May 2002, modifications for the strength of organic soils from data provided by DWR, and workshops held during the current study. A summary of the material properties used in the analyses is shown in Table 2-6. The typical location of the materials is shown on Figures 2-1 and 2-2.

For this study, it was assumed that peat under levee conditions applied to all peat that is located below a line projected along both the slough side and reservoir side levee slope through the peat to the underlying sand as shown on Figure 2-1. All peat outside of this limit was considered to be free field peat.

The strength of new embankment materials was reduced from shear strengths normally assigned for engineered sandy fills to account for shearing and cracking within the embankment fill during consolidation of the underlying peat and subsequent deformation of the new fill.

For the loose upper portion of the silty sand layer that exists in some portions of the islands, the post-liquefaction undrained residual shear strength was taken as 200 psf, based on the average estimated corrected penetration resistance (SPT). For portions of the island where this loose layer does not occur, this soil layer was assumed to have the same shear strength as the underlying sand.

An undrained shear strength of 200 psf, similar to that of the loose sand in the upper part of the silty sand layer, was assumed for evaluating the effect on stability for sandy portions of the existing levees on the reservoir side where the fill may be loose due to placement. No data from specific investigations for the density of the existing sandy levee fill were available for review.

2.2.3.2 Permeability

Generally, the coefficient of permeability for the various layers are the same as used in URS (2000). The coefficient of permeability for the peat material and the underlying sand previously used in the URS (2000) analysis were reviewed for the present study.

The coefficient of permeability for peat was estimated using the correlation relationship based on void ratio as proposed by W. Dhowian and T.B. Edil (1980). A typical void ratio of 0.7 was estimated based on several laboratory test results performed by HLA in 1989. The ratio of anisotropy for peat was estimated using the relationship proposed by W. Dhowian and T.B. Edil (1980).

The coefficient of permeability for the underlying sand was reviewed by comparing the value previously used with values estimated using relationships correlating grain size distribution and permeability (e.g., Cederegren 1989 and Sherard 1984). The correlations proposed by Cederegren and Sherard relate permeability to D_{10} and D_{15} , respectively. The average values of D_{10} and D_{15} for the aquifer were selected based on several gradation test results performed by HLA (1989) in Webb Tract and Bacon Island. The estimated permeabilites were in close agreement with the value used in our previous analyses.

Values for the coefficient of permeability used in the seepage analyses are listed in Table 2-7.

2.2.4 Reservoir Stage and Slough Water Level

At each section and case analyzed, a combination of reservoir and slough water surface levels that produce critical conditions was used. A high slough water surface elevation, combined with a low reservoir elevation, is potentially the most critical to the island-side slope. A low slough water surface elevation, combined with a high reservoir elevation, is potentially the most critical to the slough-side slope.

2.2.4.1 Reservoir Stage

The reservoirs will operate at various levels during a typical calendar year. Patterns for reservoir levels were developed through operation studies, as reproduced on Figure 2-5. In a typical year, for a little less than two months (May and June), the reservoirs will be at their maximum operating water level (+4). During about five months (September through January of the following year), the reservoirs will be at their lowest operating level or will be empty. Figure 2-5 shows an intermediate constant reservoir stage at about –11 in the second week of February to the third week in March. In between these three periods of time, variations of the reservoir level will be approximately linear.

The maximum and minimum levels for the reservoir last for extended periods of time defining conditions that correspond to normal operation. Depending on the analysis case considered, the maximum or minimum level may be the most critical for a given slope and analysis case. The most critical of either the maximum or minimum reservoir levels were considered in these analyses.

2.2.4.2 Slough Water Level

Slough water levels vary with tide cycles and flooding events.

For the analysis of the long-term condition of the reservoir-side slope, it was assumed that the water level in the slough could reach peak flood level at least once during the design life of the reservoir. The maximum peak flood elevation corresponding to the 100-year flood condition is +7.2 feet at Bacon Island and +7.0 feet at Webb Tract (Flooding Analysis, URS 2003). For the current study, a maximum peak flood elevation of +7.0 feet was used.

The sudden drawdown condition does not represent a “normal” condition. Therefore, it was combined with a flood condition less demanding than considered for the long-term condition. For the sudden drawdown analysis case, a slough water elevation of +6 feet was used. This elevation was selected from a review of gauge recordings and historical data applicable to the two sites. In these data, it was noted that the maximum peak flood occurs over a very short time, and hence should not lead to a steady-state condition during the relatively short duration of the sudden drawdown. The selected “sustained” flood elevation of +6 feet conservatively represents a critical condition for this analysis case.

For the stability evaluation of the slough-side slopes, the water surface level in the slough at an average low tide elevation (-1 feet) was used. This represents a reasonably conservative condition. Seismic conditions were analyzed for slough water levels corresponding to high (+3.5 feet), average (+1.5 feet), and low (-1.0 feet) tides.

The water elevations discussed above are tabulated along with the results of the stability analyses in Section 4.

2.3 ANALYSIS CRITERIA

2.3.1 Seepage Analysis

The evaluation criterion established by USACE (1997) was used in this project to determine whether seepage mitigation measures are needed or not. This evaluation criterion is based on the exit gradient at or near the toe of levee; the maximum acceptable gradient is 0.3.

2.3.2 Stability Analysis

Because critical conditions may arise either on the slopes facing the slough side or the reservoir side, the factors of safety of both slopes were assessed. The following analysis conditions were evaluated.

2.3.2.1 End-of Construction

The end-of-construction scenario is the condition occurring immediately after placement of new fill on the reservoir island side of the levee. Fill is placed in thin layers and compacted. Immediately after fill placement, relatively impervious materials such as peat and clay in the levee and foundation will not have had sufficient time to dissipate construction-induced excess pore pressures. Hence, at the end of construction, undrained shear strengths are normally used to characterize the cohesive soils of the levee and foundation.

End-of-construction stability should be evaluated to check whether the levee strengthening could be constructed in a single stage without any excessive undrained shear deformation. If the undrained shear strength of the peat is not sufficient to provide short-term stability, the placement of levee-strengthening fill will have to be done in several stages. This will require limiting the maximum height of fill for each stage and waiting for a sufficient time to let the peat consolidate and gain its strength before continuing another fill placement. This issue is addressed in more detail in the Earthwork Construction Cost Estimate report (URS, 2003).

2.3.2.2 Long-Term Operation

The analysis of long-term levee stability involves the post-construction conditions when strength gain has occurred, and normal operation of the reservoir is in place. Two combinations of water levels (high reservoir and low slough water, and vise-versa) on the reservoir and slough sides were selected to produce the most critical load cases that could be encountered during such operation.

As discussed in Section 2.2.2, the deformed geometry due to the consolidation of peat under the proposed fill was used to assess the factor of safety for the long-term slope stability condition.

2.3.2.3 Sudden Drawdown

The sudden drawdown case affects the reservoir-side slope when the reservoir water level drops rapidly. Such condition may result from emergency drainage of the reservoir.

Because the drop in reservoir level can occur at a relatively rapid rate, the peat and other fine-grained soils would not have enough time to drain, and undrained strengths after long-term consolidation are used in the analysis. The three-stage stability computations (Duncan et al. (1990) incorporated in the computer program UTEXAS3 was the methodology used in rapid drawdown stability analysis.

2.3.2.4 Pseudo-Static Analysis

Pseudo static analysis is used to estimate the yield acceleration (K_y). The use of the calculated yield acceleration to estimate earthquake-induced deformation of the levee systems is discussed in the Seismic Analysis Report. Water levels on the island and slough sides were selected to produce critical cases. The strength of soil layers that are potentially liquefiable were taken as the undrained residual shear strength (Seed and Harder, 1990). Yield accelerations were computed for the most critical failure surfaces.

Undrained shear strengths in potentially liquefiable soils were also used in computing post-seismic stability. The two-stage stability computations (Duncan et al., 1990) incorporated in the computer program UTEXAS3 was the methodology used in post-seismic stability analysis.

2.3.2.5 Evaluation Criteria

Criteria for the calculated factors of safety for each case of static stability are summarized in Table 2-8. These selected factors of safety are based on the significance of the project; the consequences of failure; uncertainties in estimated parameters; cases considered; and criteria from several agencies.

Table 2-1 – Island Perimeter Subdivision (Bacon Island)

Sections	Start Station	End Station	Section Length (ft)	El. Bottom of Peat (ft)	El. Top of Peat (ft)	Existing Levee Crest El. (ft)	Reference Number or Source of Information
Section 1	720+00 400+00 545+00	30+00 490+00 580+00	6,636 9,000 3,500	- 20	0	+8.0	Borings & CPT Logs Profile Crest El. From URS 2000 analyses
Section 2	350+00	400+00	5,000	-25	-5	+8.0	Borings & CPT Logs Profile Crest El. From URS 2000 analyses
Section 3	30+00 210+00 490+00 580+00	170+00 350+00 545+00 720+00	14,000 14,000 5,500 14,000	- 30	0	+8.0	Borings & CPT Logs Profile Crest El. From URS 2000 analyses
Section 4	170+00	210+00	4,000	- 40	0	+8.0	Borings & CPT Logs Profile Crest El. From URS 2000 analyses

Table 2-2 – Island Perimeter Subdivision (Webb Tract)

Sections	Start Station	End Station	Section Length (ft)	El. Bottom of Peat (ft)	El. Top of Peat (ft)	Existing Levee Crest El. (ft)	Reference Number or Source of Information
Section 1	610+00 510+00	200+00 540+00	28,247 3,000	- 25	-5	+8.0	Borings & CPT Logs Profile Crest El. from URS 2000 analyses
Section 2	200+00 430+00 460+00 540+00	390+00 450+00 510+00 610+00	19,000 2,000 5,000 7,000	- 35	-5	+8.0	Borings & CPT Logs Profile Crest El. from URS 2000 analyses USBR DW Project, status rpt 8/3/01
Section 3	390+00	430+00	4,000	- 40	0	+8.0	Borings & CPT Logs Profile Crest El. from URS 2000 analyses
Section 4	450+00	460+00	1,000	0	See Section 2.2.1	+8.0	Borings & CPT Logs Profile Crest El. from URS 2000 analyses

Table 2-3 – Existing Section Geometry (Bacon Island)

Sections	Reservoir (island) Bottom El. (ft) [2]	Island Side Slope Angle (°) [1]	Average Slough Slope Angle (°) [1]	Steepest Slough Slope Angle (°) [1]	Flattest Slough Slope Angle (°) [1]	Average Crest Width (ft) [2]	Average Slough Bottom El. (ft) [2]	Highest Slough Bottom El. (ft) [2]	Lowest Slough Bottom El. (ft) [2]
Section 1	-9	16 (3.5:1)	19 (2.9:1)	30 (1.7:1)	11 (5.1:1)	26	-22	-14	-30
Section 2	-10	14 (4.0:1)	19 (2.9:1)	22 (2.5:1)	16 (3.5:1)	26	-26	-20	-30
Section 3	-9	15 (3.7:1)	20 (2.7:1)	30 (1.7:1)	15 (3.7:1)	26	-22	-10	-34
Section 4	-9	18 (3.1:1)	18 (3.1:1)	21 (2.6:1)	15 (3.7:1)	28	-31	-29	-33

Notes: [1] Slope angles are measured with respect to horizontal, and expressed as horizontal to vertical

[2] Elevations and widths are based on topographic maps by MBK Engineers (Jan.,96) for Bacon Island and Murray, Burns & Kielen (April,96) for Webb Tract

Table 2-4 – Existing Island Geometry (Webb Tract)

Sections	Reservoir (island) Bottom El. (ft) [2]	Island Side Slope Angle (°) [1]	Average Slough Slope Angle (°) [1]	Steepest Slough Slope Angle (°) [1]	Flattest Slough Slope Angle (°) [1]	Average Crest Width (ft) [2]	Average Slough Bottom El. (ft) [2]	Highest Slough Bottom El. (ft) [2]	Lowest Slough Bottom El. (ft) [2]
Section 1	-9	15 (3.7:1)	23 (2.4:1)	35 (1.4:1)	13 (4.3:1)	20	-23	-15	-30
Section 2	-9	13 (4.3:1)	19 (2.9:1)	27 (2.0:1)	12 (4.7:1)	20	-27	-13	-50
Section 3	-8	13 (4.3:1)	20 (2.7:1)	24 (2.2:1)	16 (3.5:1)	17	-21	-13	-33

Notes: [1] Slope angles are measured with respect to horizontal, and expressed as horizontal to vertical

[2] Elevations are based on topographic maps by MBK Engineers (Jan., 96) for Bacon Island and Murray, Burns & Kielen (April,96) for Webb Tract

Table 2-5 – Compressibility Parameters for Peat

Material	C_C	CR	e_0	C_α	c_v (ft^2/yr)	OCR
Peat Under Levee	5.0	0.55	8.1	0.17	75	1.0 to 1.5
Free Field Peat	3.7	0.51	6.3	0.17	75	1.5 to 2.0

Table 2-6 - Material Properties

Material	Weight, γ, lb/ft^3		Undrained Shear Strength, lb/ft^2	Effective Strength		Total Strength	
	wet	Sat.		φ', degree	C', lb/ft2	ϕ_t, degree	C_t, lb/ft2
Rock Fill	140	140		40	0	40	0
New Fill [1]	110	120		30	0	30	0
Existing fill sand	110	110		30	0	30	0
Existing fill, sand with clay and peat	110	110		30	0	30	0
Peat under dam [2]	70	70	450	28	50	17	100
Free field peat [2]	70	70	200	20	50	13	100
Deep sand		125		36	0	36	0
Gray fat clay		85	250	25	0	30	100

[1] New fill shear strength properties are reduced to account for shearing within the embankment during consolidation of the underlying peat and subsequent deformation of the new fill.

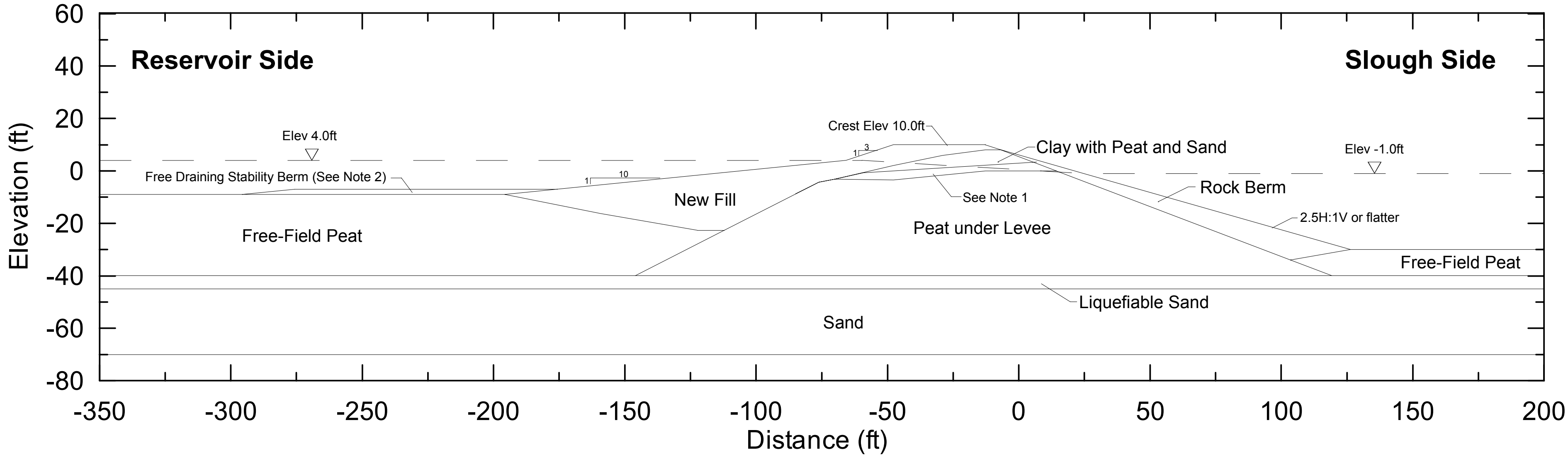
[2] Peat shear strength values (provided by DWR on 9/30/02) are based on back calculations and data for similar islands.

Table 2-7 - Permeability of Soils Used in Seepage Analysis

Material	Vertical Permeability Ky (cm/s)	Horizontal Permeability Kx (cm/s)	Ky/Kx
Existing Sandy Fill (with clay and peat)	1×10^{-5}	1×10^{-4}	0.1
Existing Clayey Fill (Bay Mud)	1×10^{-7}	1×10^{-6}	0.1
Peat	1×10^{-6}	2×10^{-4}	0.005
Sand	1×10^{-4}	1×10^{-3}	0.1
Clay	1×10^{-6}	1×10^{-6}	1
Planned Fill (sand)	1×10^{-3}	1×10^{-3}	1

Table 2-8: Minimum Factors of Safety for Static Stability

Case	Material Properties	Phreatic Surface	Minimum Factor of Safety
End of Construction	Unconsolidated undrained shear strength	Construction-induced excess pore pressures with high and low river elevations	1.3
Sudden Drawdown	Consolidated undrained shear strength	Rapid Drawdown from normal pool to dead storage with low river elevation (use phreatic surface from steady-state seepage with surface following the island slope.	1.2
Steady-State Seepage	Consolidated drained shear strength	Steady-state seepage under normal pool with low river elevation	1.5
Seismic - Post Liquefaction Stability	Consolidated Undrained -Based on SPT	Steady-state	1.1



Note 1: Portion of existing levee assumed to be potentially liquefiable.
Note 2: Free draining stability berm required to meet long term stability condition where base of peat is -30 feet or lower.

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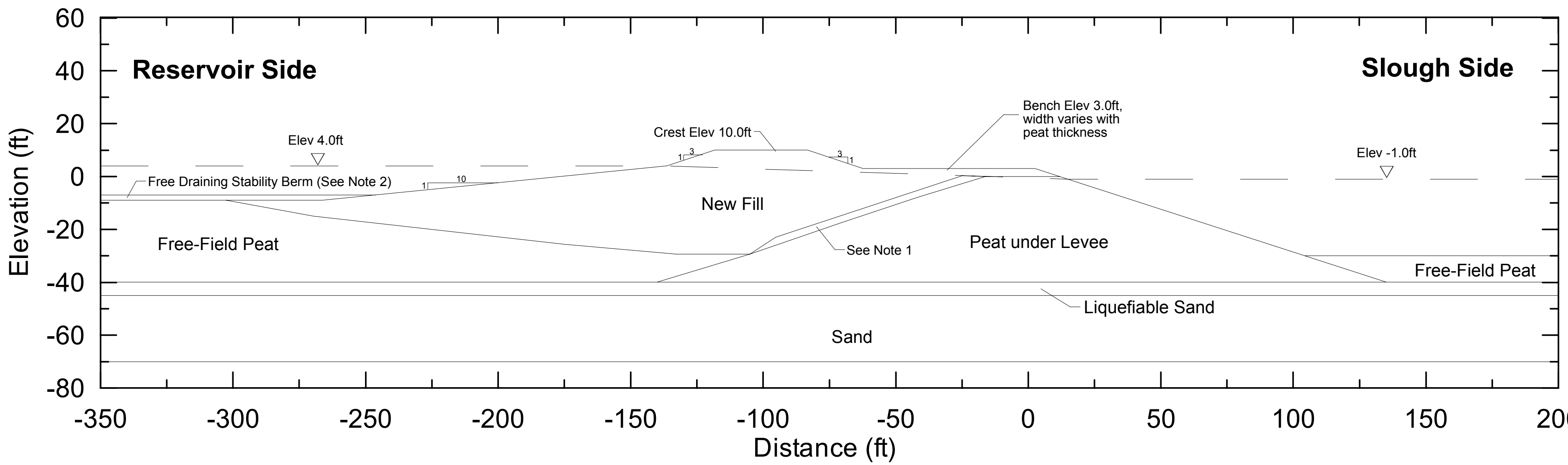
IN-DELTA STORAGE PROJECT	TYPICAL SECTION FOR STABILITY ANALYSIS "ROCK BERM" OPTION	Figure 2-1
URS — April 2003 Project # 26814103		



Note 1: Portion of existing levee assumed to be potentially liquefiable.
Note 2: Free draining stability berm required where liquefaction of upper sand occurs and base of peat is -40 feet or lower.

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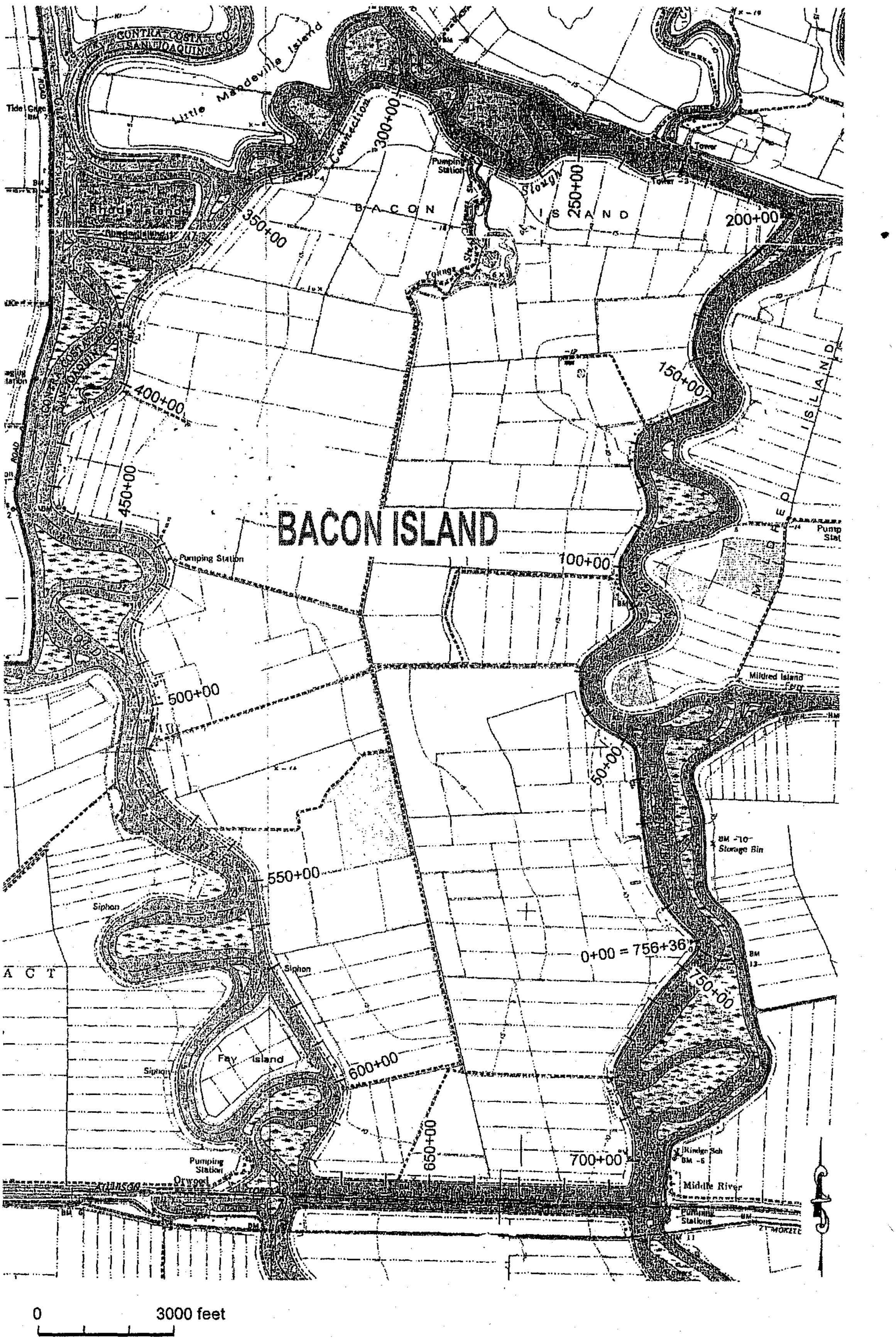
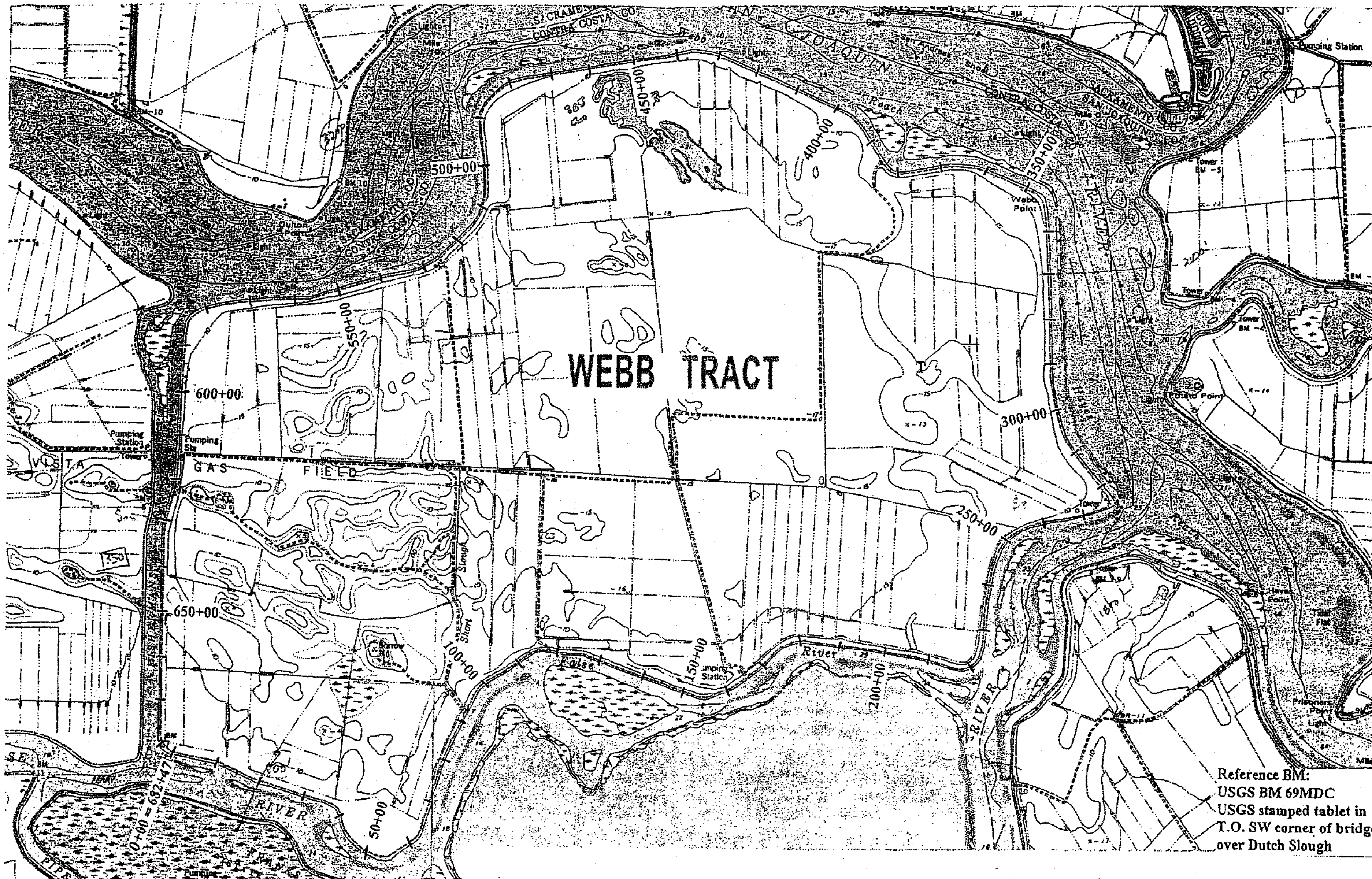
IN-DELTA STORAGE PROJECT		TYPICAL SECTION FOR STABILITY ANALYSIS "BENCH" OPTION	Figure 2-2
URS	April 2003 Project # 26814103		



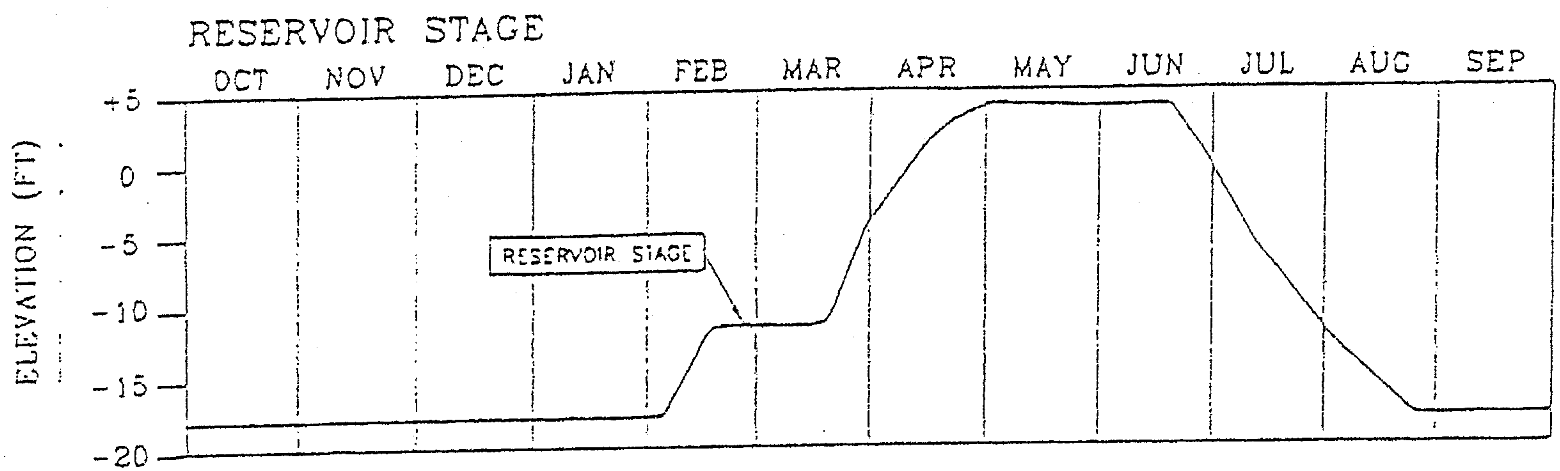
URS	Project No. 26814104	EMBANKMENT STATIONS AT BACON ISLAND RESERVOIR	Figure 2-3
	WEBB Tract Reservoir		



0 3000 feet

URS

Project No. 26814104	EMBANKMENT STATIONS AT WEBB TRACT RESERVOIR	Figure 2-4
WEBB Tract Reservoir		



Source: from Hulgren, 1997a

IN-DELTA STORAGE PROJECT		RESERVOIR STAGE CURVE	Figure 2-5
URS	December 2002 Project # 26814103		

3.1 GENERAL

Seepage analyses for the reservoirs was previously performed as described in URS (2000). The sections analyzed in URS (2000) were reviewed and determined to be appropriate for the current study. The primary change in the current study is a reduction in the normal operating reservoir water elevation from +6.0 feet to +4.0 feet.

3.2 METHODOLOGY

The computer program SEEP/W (Geo-Slope International Ltd., 1994) was used to estimate seepage conditions through transverse sections of the existing levees at Webb Tract and Bacon Island. SEEP/W uses a two-dimensional finite element method to model seepage conditions and assumes that flow through both saturated and unsaturated media follows Darcy's Law. The seepage analyses were conducted considering steady-state conditions.

Using the SEEP/W mesh generation program, finite element meshes were generated to model the multiple seepage conditions considered for the levees on Webb Tract and Bacon Island. The element material types are represented in the models as different colors, as shown on Figure 3-1. Fixed boundary conditions were used to model constant reservoir and slough heads, heads within pumping wells and far-field groundwater levels. Other portions of the levee and ground surfaces on the islands were modeled using an unrestricted, free-flowing boundary condition; that is, a boundary condition that is determined at each node by SEEP/W during the analysis of flow conditions. The bottoms of the cross-sections were modeled as no-flow boundaries.

The SEEP/W analysis program was used to evaluate the steady-state phreatic surface location, the head distribution throughout the model and flow quantities at particular locations. The SEEP/W contouring program was used to generate head distribution diagrams. Phreatic surfaces, total head contours (in feet of water) and flux quantities (in gallons per minute per foot width of levee) are presented on each of the figures presenting the analysis results for each section. The flux quantities represent the flow quantity across the length of a particular flux section, which is symbolized as an arrow on the figures.

3.3 ANALYSES SECTIONS

Three sections were considered for the seepage analysis, two at Webb Tract and one at Bacon Island. The sections at Webb Tract were selected at Stations 630+00 (Figure 3-1) and 260+00 (Figure 3-3) to represent the narrow (400 feet) and wide (1,200 feet) slough, respectively. The section at Bacon Island was selected at Station 665+00 (Figure 3-10) to represent an average slough width (700 feet), which is more common around the islands.

Previous analyses (URS, 2000) considered Bacon Island Station 220+00 as a critical section representing a narrow slough (450 feet) for Bacon Island. For the purpose of this analysis, Webb Tract Station 630+00 is considered to be representative of the narrow slough for the two islands.

In addition to the above three analyses sections, we have also evaluated the two sections at Webb Tract, assuming the sand is exposed in the island interior (Figures 3-4 and 3-8).

3.4 ANALYSIS CONDITIONS

For each section, three seepage conditions were evaluated: (1) existing conditions, (2) full reservoir with no pumping at the interceptor wells, and (3) full reservoir with required pumping at the interceptor wells. Existing conditions were first analyzed to evaluate the pre-reservoir seepage conditions. Full reservoir conditions without underseepage remediation were analyzed as an intermediate condition to estimate the impacts of the reservoirs on neighboring islands. Full reservoir conditions with pumping at the interceptor well system were analyzed to evaluate the efficiency of the proposed interceptor well system and to estimate the minimum pump rate (in gallons per minute per foot of levee) required to reestablish pre-reservoir seepage conditions at the far levee.

3.5 BOUNDARY CONDITIONS

The primary boundary conditions affecting the seepage models include the constant head boundaries imposed by presence of the slough, the full reservoir, and the groundwater conditions within the adjacent island. The slough was modeled as having a constant elevation head of –1.0 feet (using the USGS elevation datum). The slough level at the islands varies up to about three and a half feet between daily high and low tides, however the low tide value of -1 feet was considered for comparison with the full reservoir case. For the full reservoir condition, a constant normal operating reservoir water level of +4 feet was used, based on current understanding of expected reservoir operation levels. Sensitivity with respect to slough water level was analyzed for Webb Tract station 630+00 using a high tide (+3.5 feet) and full reservoir conditions. The cross sections considered for seepage analysis together with water elevations used in both reservoir and slough sides are summarized in Table 3-2.

The far-field boundary condition at the neighboring island under existing conditions was estimated using a groundwater level at about 2 feet below the average ground elevation of the island.

For the full reservoir condition with pumping at the interceptor wells, a constant flow boundary was placed through the sand aquifer at the location of the well line. This boundary condition was used to represent the average flow rate along the well line during pumping, and was varied until the pre-reservoir conditions were re-established.

3.6 RESULTS

The analysis results are summarized for each case in Table 3-3. The table presents the following:

- The average total head (in feet) in the sand aquifer at the near levee (Webb Tract or Bacon Island) centerline.
- The average total head (in feet) in the sand aquifer at the far levee (adjacent island) centerline.
- The flow rate through the sand aquifer at the far levee centerline.
- Exit gradient at the land side toe of the far levee.
- The corresponding pump rates for individual interceptor wells spaced at 160 and 200 feet.

Discussion of the findings for each cross-section are presented below.

Webb Tract Station 630+00. This cross-section was considered to be a critical seepage condition for Webb Tract, as the adjacent island levee is only about 400 feet away (center to center).

The total head within the sand aquifer at each levee under existing seepage conditions is about –12 feet, as shown on Figure 3-1. The existing conditions diagram shows a significant head loss within the channel peat, indicating the importance of the channel peat's influence on the seepage rates under the levees.

Under full reservoir conditions with no seepage remediation, there is about a five-foot increase in the total head beneath the far levee, as shown on Figure 3-2. In addition, a review of the exit gradients near the toe of the far levee indicates an increase from 0.2 under existing conditions to 0.6 under full reservoir conditions. The exit gradients shown under full reservoir conditions have the potential to cause sand boils and piping of levee material (USACE, 1997) on the neighboring island and seepage flow will increase by about 2.5 times. Using interceptor wells, the minimum pumping rate needed to re-establish pre-reservoir conditions at the adjacent island is about 6 gpm for wells spaced at 160 feet and 7.5 gpm for 200 feet.

Sensitivity under high tide (+3.5 feet) and full reservoir conditions was also checked for this cross-section for comparison with the low tide and full reservoir conditions discussed above. The total head within the sand aquifer at each levee under existing seepage conditions is at about elevation –12 feet, as shown on Figure 3-3. Under full reservoir conditions with no seepage remediation, there is about a five-foot increase in the total head beneath the far levee, as shown on Figure 3-3. In addition, a review of the exit gradients near the toe of the far levee indicates an increase from 0.2 under existing conditions to 0.6 under the full reservoir case.

For conditions where the sand aquifer is exposed within Webb Tract near the new embankment with no seepage remediation, there is a six-foot increase in the total head beneath the far levee (Figure 3-4). In addition, a review of the exit gradients near the toe of the far levee indicates that gradients of 0.7 exist at the ground surface under full reservoir conditions. Under gradients of this magnitude, there would likely be sand boils and piping of levee material on the neighboring island. Under full reservoir conditions with pumping at the interceptor wells, the minimum pump rate needed to re-establish the pre-reservoir conditions at the adjacent island is about 8.7 gpm for wells spaced at 160 feet and 10.8 gpm for wells spaced at 200 feet (Figure 3-5).

Webb Tract Station 260+00. This cross-section was considered to be one with the widest slough (1200 feet from center to center of levees). The total head within the sand aquifer at each levee under existing seepage conditions is about –12 feet, as shown on Figure 3-6. The existing conditions diagram shows a significant head loss within the channel peat, indicating the importance of the channel peat's influence on the seepage rates under the levees.

Under full reservoir conditions with no seepage remediation, there is about a two foot increase in the total head beneath the far levee, as shown on Figure 3-7. In addition, a review of the exit gradients near the toe of the far levee indicates an increase from 0.1 to 0.2 from the exiting condition to the full reservoir case, respectively. Under full reservoir, these gradients would not likely cause sand boils or piping of levee material on the neighboring island. However, seepage flows could increase by about 1.6 times. Under full reservoir conditions with pumping at the interceptor wells, the minimum pump rate needed to re-establish pre-reservoir conditions at the adjacent island is estimated to be about 5.7 gpm for wells spaced at 160 feet and 7.2 gpm for wells spaced at 200 feet.

For conditions where the sand aquifer is exposed within Webb Tract near the new embankment with no seepage remediation, there is a three foot increase in the total head beneath the far levee (Figure 3-8). In addition, a review of the exit gradients near the toe of the far levee indicates that under full reservoir, the exit gradient is about 0.2. These gradients would not likely cause sand boils or piping of levee material on the neighboring island. However, seepage flows would increase by about 1.6 times. Under full reservoir conditions with pumping at the interceptor wells, the minimum pumping rate needed to re-establish the pre-reservoir conditions at the adjacent island is about 8.8 gpm for wells spaced at 160 feet and 10.9 gpm for wells spaced at 200 feet (Figure 3-9).

A sensitivity analysis allowing the water level to vary from –1 feet to +3.5 feet in elevation showed an insignificant difference between the two cases.

Bacon Island Station 665+00. This cross-section was considered to be an average representative slough width of 700 feet from center to center of levees. This condition will exist in both islands. The total head within the sand aquifer at each levee under existing seepage conditions is about –12 feet, as shown on Figure 3-10.

Under full reservoir conditions with no seepage remediation, there is about a two and one-half foot increase in the total head beneath the far levee, as shown on Figure 3-11. In addition, a review of the exit gradients near the toe of the far levee indicates an increase from 0.20 to 0.30 from the exiting condition to the full reservoir case, respectively. Under full reservoir, these gradients would not likely cause sand boils or piping of levee material on the neighboring island. However, seepage would increase by about two times. Although not calculated, we estimate that the minimum pumping rate needed to re-establish pre-reservoir conditions at the adjacent island to be about 6 gpm for wells spaced at 160 feet or about 7.5 gpm for wells spaced at 200 feet.

3.7 SEEPAGE CONTROL ALTERNATIVES

Potential seepage control measures for the In-Delta-Storage islands include interceptor wells, slurry cut-off walls, reservoir floor clay blanket, and collector trenches/French drains in the neighboring islands among others. These techniques vary in cost, constructibility, feasibility, and operation and maintenance. A brief discussion of these alternatives is presented below. The advantages and limitations of each method are discussed below.

3.7.1 Interceptor Wells

This solution relies on a series of active extraction wells located on the crest of the reservoir islands' embankments. The wells are actively operated to draw the aquifer down such that seepage flows in the neighboring islands are maintained to the same levels as before reservoir island project is implemented. This solution would require well spacing varying from 160 feet (along sections with thinner peat layers and narrower sloughs) to 200 feet or greater in spacing (for thicker peat layers and wider sloughs). Assuming a 200-foot well spacing is an average representative well spacing, the pumping rate to re-establish the existing condition (pre-project) would be about 8 gpm per well. The excess seepage flow into the neighboring islands, absent any pumping, would be on average 2 to 4 gpm per 100-foot section of levee.

The major limitations associated with active pumping to control seepage are the required operation and maintenance to keep the interceptor wells in good operating conditions. The maintenance items would include regular cleaning, surging, and disinfecting of each well at 2 to 5 year intervals. Often, well efficiency decreases in time because of excessive siltation and clogging, biological fowling and chemical encrustation. At that point, wells would need to be replaced or cleaned out. Based on general experience, one can expect that 50 percent of the wells would be replaced every 50 years. Because extraction wells may cause migration of fines, the proper well design and construction would be needed to minimize desitling the aquifer. In conjunction with these potential problems, periodic monitoring of well performance and surveying for subsidence are required.

3.7.2 Slurry Wall

The slurry cut-off wall is one of the most common solutions used for under-seepage control. It is a passive solution that requires no maintenance. However, considering the soft nature of the peat layers within the existing levee and foundation, the construction of slurry cut-of walls could become challenging because of the potential squeezing soft strata within the slurry trench. Experience with slurry walls along flood control levees in the Sacramento region has often resulted in leaks of slurry during construction. Because of the potential challenges associated with the construction of this technique, test sections would need to be conducted to validate the feasibility and constructability of slurry walls in the Delta.

Compared to the interceptor wells solution, the slurry cut-off method could be as much as 2 to 3 times more expensive. These cost comparisons are based on rough unit prices of similar constructed projects in the region.

3.7.3 Reservoir Floor Clay Blanket

The reservoir floor clay blanket is considered for comparison purpose. A 1000-foot long (minimum distance from the toe of the embankment) and three-foot thick clay blanket would be needed from the toe of the embankment to provide under-seepage control. Although this solution also offers a passive seepage control measure that would not require operation and maintenance, it could however, be exposed to the potential risks of drying and cracking if not maintained continuously under water.

This method would require a large volume of imported clay. The cost of such solution could be as high as six times that of the interceptor wells.

3.7.4 Collector Trench

Collector trenches constructed at the landside toe of the adjacent levees would be an effective method of collecting excess seepage and protecting against piping due to high exit gradients. The collector trenches would penetrate the overlying peat to the underlying sand aquifer. Trench backfill and collector pipes within the trench would be designed to meet filter criteria (USACE, 2000). This alternative is also a passive seepage control system and would not require operation and maintenance. Because of the proposed maximum reservoir elevation of +4 feet (as opposed to +6 feet from previous studies), the excess flow caused by the reservoir is smaller (2 gpm for +4 feet vs. 4 gpm for +6 feet). The excess seepage would therefore be accommodated by discharging flows from the collector trench into the local drainage ditches within the neighboring islands. This technique is highly effective and readily constructed.

The major limitation of such a solution is the requirement for an encroachment permit within the neighboring islands and for a long term agreements with the neighboring island owners (including possibly some cost sharing of pumping effort to drain the islands). In other words, the seepage control measure would not be on State owned land, and hence access could become an issue.

The collector trench solution is one of the most attractive of the four on the basis of engineering merits only. It comes at the lowest cost among the four alternatives, and is approximately one third the cost of the interceptor wells.

3.8 SUMMARY OF FINDINGS

The seepage analyses conducted for three cross sections taken along the Webb Tract and Bacon Island levees shows that the proposed reservoir islands may increase the water table beneath the levee at adjacent islands 2 to 3.5 feet, and that flooding may occur in the neighboring islands in the absence of a seepage control system. Seepage flows at the neighboring island will increase by 1.5 to 2.5 times for an operating reservoir level of +4 feet. Exit gradients will also increase with greater increases where slough widths between the reservoir and the adjacent islands are narrower. At the narrowest section analyzed (Webb Tract Section 630+00) exit gradients increase to levels that could cause sand boils and piping.

A properly functioning seepage control system can be used to minimize the effects of the proposed reservoirs on adjacent islands, including the potential for rises in the ground water table or flooding. Interceptor wells are recommended for seepage control based on cost for alternatives that can be constructed within the reservoir areas. In order for the well system to intercept the reservoir-induced seepage and maintain existing seepage conditions beneath the levees at adjacent islands, pump rates of about 6 to 8 gpm (for wells at 160-foot spacing) would be required.

For both Webb Tract and Bacon Island, the interceptor well system should extend to the bottom of the sand aquifer. The pumping well should be screened over the entire length of the aquifer to achieve the required drawdown at the well, and the pumps should efficiently handle the required pump rate. A spacing of 160 feet between pumping wells appears to be adequate; however, optimum spacings and pump rates may be found for each levee section during design of the project. Following detailed investigations of subsurface conditions, adjustments in the well interceptor system design will be required to accommodate varying conditions, ranging from areas where little or no pumping may be needed (e.g., next to the San Joaquin River) to areas where pumping rates may be much higher than is typical (e.g., along localized gravelly portions of the aquifer).

The interceptor well concept generally appears to be able to mitigate seepage problems induced by the proposed reservoirs. Proper design, construction, and maintenance will be key to the success of the interceptor well system. The water table level on the adjacent islands is considered to be an important indicator of impacts detrimental to those islands, as a significant rise in the ground water table may affect agricultural operations and production rates. The wells will have to be maintained at regular intervals to ensure their effectiveness. Further, observation wells installed on the adjacent island levees must be monitored consistently so that the interceptor wells are operated at the pumping rate that minimizes potential impacts on neighboring islands.

Table 3-1 – Soil Properties Used in Seepage Analysis

Cross Section	Soil Layer	Approximate Soil Layer Thickness (feet)	Horizontal Hydraulic Conductivity K_x (cm/s)	Vertical Hydraulic Conductivity K_y (cm/s)
Webb Tract Sta. 260+00	Fill Material [1]	12	1×10^{-4}	1×10^{-5}
	Peat	32	2×10^{-4}	1×10^{-6}
	Sand	40	1×10^{-3}	1×10^{-4}
	Lower Clay	--	1×10^{-6}	1×10^{-6}
	New Fill (Sand)	Varies	1×10^{-3}	1×10^{-3}
Webb Tract Sta. 630+00	Fill Material [2]	10	1×10^{-4}	1×10^{-5}
	Fill Material [3]	5	1×10^{-6}	1×10^{-6}
	Peat	20	2×10^{-4}	1×10^{-6}
	Sand	45	1×10^{-3}	1×10^{-4}
	Lower Clay	--	1×10^{-6}	1×10^{-6}
	New Fill (Sand)	Varies	1×10^{-3}	1×10^{-3}
Bacon Island Sta. 665+00	Fill Material [4]	20	2×10^{-4}	1×10^{-6}
	Peat	18	2×10^{-4}	1×10^{-6}
	Sand	22	1×10^{-3}	1×10^{-4}
	Lower Clay	--	1×10^{-6}	1×10^{-6}
	Channel Silt	3	1×10^{-6}	1×10^{-6}
	New Fill (Sand)	Varies	1×10^{-3}	1×10^{-3}

[1] Clay with Peat and Sand
[2] Sand
[3] Clay
[4] Peat

Table 3-2 – Cross Sectional Models Used in Seepage Analaysis

Cross Section	Water Elevation Slough (feet)	Water Elevation Reservoir (feet)
Webb Tract Sta. 260+00	-1 -1	Empty +4
Webb Tract Sta. 630+00	-1 -1 +3.5 [1] +3.5 [1]	Empty +4 Empty +4
Bacon Island Sta. 665+00	-1 -1	Empty +4

[1] Average high tide used. Reservoir full stage does not correspond to highest water stages typically between December through February.

Table 3-3 – Seepage Analysis Results

Location	Condition	Head in Sand at Near Levee CL (feet)	Head in Sand at Far Levee CL (feet)	Flow rate at Far Levee CL (gpm/ft)	Exit Gradient at Far Toe of Far Levee	Pumping Rate Required For Wells (gpm)	
						160’ spacing	200’ spacing
Webb Tract - Station 630+00	Existing	-12	-12	0.0045	0.21	NA	NA
	full reservoir	-2	-7	0.0115	0.57	NA	NA
	full reservoir w/pumping	-11	-12		0.23	6	7.5
	full reservoir high tide +3.5	-1	-6.5		0.64	NA	NA
	full reservoir exposed sand	0	-6.5		0.64	NA	NA
	full reservoir exposed sand w/pumping	-11	-12		0.24	8.7	10.8
Webb Tract - Station 260+00	existing	-11.5	-11.5	0.0056	0.13	NA	NA
	full reservoir	0.5	-9.5	0.0090	0.24	NA	NA
	full reservoir w/pumping	-10	-11.5		0.13	5.7	7.2
	full reservoir exposed sand	1.5	-9		0.25	NA	NA
	full reservoir exposed sand w/pumping	-11	-11.6		0.13	8.8	10.9
Bacon Island - Station 665+00	existing	-10.5	-10.5	0.0032	0.23	NA	NA
	full reservoir	0	-8	0.0067	0.34	NA	NA

4.1 METHODOLOGY

The stability of the embankments were analyzed using the limit equilibrium method based on Spencer's procedure as coded in the computer program UTEXAS3 (Wright (1992)). In Spencer's procedure, side forces acting on all slice interfaces are assumed to have the same inclination. The trial-and-error solution coded in the program involves successive assumptions for the factor of safety and side force inclination until both force and moment equilibrium conditions are satisfied. UTEXAS3 was used to compute factors of safety using either circular or general shaped, noncircular shear surfaces.

UTEXAS3 can perform two-stage and three-stage computations to simulate rapid drawdown and seismic loading. Both procedures require the input of the effective (S-envelope) and total (R-envelope) strength envelopes. Two-stage stability computations are appropriate for earthquake loadings and the three-stage computations are appropriate for sudden drawdown especially for materials, which may dilate and become weaker as they drain (Duncan et al. (1990)).

For the end-of construction case, the peat strengths were taken as the undrained shear strength shown in Table 2-6.

For psuedo-static analysis, liquefaction potential and post-liquefaction residual shear strengths were developed to support the calculation of the yield accelerations. The potential for liquefaction was estimated based on the mean corrected "clean sand" blowcount $N_1(60)_{cs}$ for a layer identified as prone to liquefy in the review of the subsurface conditions along the perimeter of the islands. Where liquefaction was considered to be likely, the residual undrained shear strength (S_r) of a "liquefied" zone was taken in the lower half of the estimates provided by the upper and lower bound relationships between S_r and $N_1(60)_{cs}$, published by Seed and Harder (1990).

Phreatic surfaces on the reservoir side of the embankments were assumed to be at the ground surface for analysis when the reservoir was empty. Phreatic surfaces through the embankments were assumed to be a straight line between the assumed water surface on the reservoir and slough sides of the embankment.

4.2 RESULTS

4.2.1 End-of-Construction

An analysis reflecting end-of-construction conditions was conducted for the "rock berm" option using the most critical case (base of peat at –40 feet). Slough water and reservoir groundwater levels were selected to assess a critical condition.

The analysis indicates that the height of embankment that can be constructed in a single stage is dependent on the location of the boundary between the peat under levee and free field peat. A sensitivity analysis was performed by setting the material strength for peat under the levee to be equivalent to free-field peat and observing the location of the resulting critical failure surfaces for different fill heights. The boundary between peat under levee and free-field peat can be inferred as a line approximately parallel to the reservoir-side slope and tangent to the critical failure surface as shown in Appendix A. The analysis indicates that as the location of the inferred boundary shifts towards the slough the calculated end-of-construction factor of safety is reduced. Based on the results shown in Table 4-1, first stage construction of the embankment using a 10H:1V reservoir-side slope to a height of approximately 10 feet above the reservoir bottom will result in a calculated factor of safety of 1.3, assuming that the critical failure surface for the end-of construction passes entirely through free field peat.

Based on the sensitivity analysis, embankment construction should be staged using a 10H:1V reservoir-side slope, with the first stage being no greater than 8 to 10 feet in height. Successive construction stages are assumed to be allowed after eighty percent of consolidation resulting from the previous stage of construction has occurred. Based on the rate of compression shown in Table 2-5, three months to eighteen months will be required for Case 1 and Case 2, respectively.

The above results indicate the need for careful planning and constructing the embankments in stages over several seasons (4 to 6 years). These results confirm that building up the embankments too rapidly could result in slope failure. The construction sequence of the fill in a staged fashion can be specified during design and verified during construction. This design requirement may include such criteria as minimum required factor of safety and consolidation strength gain before the next staged layer is placed.

4.2.2 Long-Term Normal Operation

"Rock Berm" Option

Analyses were performed on the slough side for the best, average, and worst slough-side slopes for Case 1 (base of peat at –20 feet) and Case 2 (base of peat at –40 feet). Results indicate that for all of the cases considered stability criteria can only be met by adding a rock berm on the slough-side toe of the existing levee. The rock berm required to meet project criteria may be reduced at some locations depending on the thickness and extent of the rockfill that exists on the slough-side slopes.

On the reservoir side, analyses were performed for the average reservoir bottom. Stability criteria was met for Case 1, but not for Case 2. The addition of a thin horizontal reservoir-side toe berm of free draining material to Case 2 was required to meet stability criteria.

The results are presented in Tables 4-2 and 4-3 along with water surface elevations assumed on either side of the embankment. Failure surfaces for all sections analyzed are included in Appendix A.

"Bench" Option

Analyses were performed using average slough-side slopes to assess what combinations of bench width and elevation proposed for the project would meet stability criteria. Critical failure surfaces passing through both the bench and the crest were considered. The analyses were performed assuming full reservoir during low tide conditions for Case 1 and Case 2. The results of the analyses are presented in Table 4-4.

Generally, higher bench elevations decrease the calculated factor of safety for surfaces assumed to pass through the crest of the embankments and increase the calculated factor of safety for surfaces passing through the bench. Increased bench widths (i.e., shifting of the embankment crest towards the reservoir) increase the calculated factor of safety for surfaces passing through the crest. Because the critical failure surfaces passing through the bench also pass through the implied embankment slope at depth they should meet long term stability criteria. The analyses indicate that bench elevations in excess of 3 feet do not meet stability criteria. Where benches having elevation of 6 feet are desired and where slough-side slopes are steeper than the average cases analyzed, rock berms should be placed on the toe of the existing levees in order to meet stability criteria.

Long term stability calculations towards the reservoir assumed a slough side bench elevation of 3 feet. Bench widths (El. +3 feet) required to meet stability criteria were 31 feet for Case 1 and 65 feet for Case 2. These bench elevations and widths were used for the remaining analyses for the "bench" option.

The results are presented in Tables 4-5 and 4-6 along with water surface elevations assumed on either side of the embankment. Failure surfaces for all sections analyzed are included in Appendix A.

4.2.3 Sudden Drawdown

"Rock Berm" Option

Computed factors of safety range from 1.6 to 1.5 for Case 1 and Case 2, respectively. These results are based on the conservative assumption that the new fill along the inside perimeter of the embankment would remain fully saturated after the occurrence of sudden drawdown; the results are summarized in Tables 4-2 and 4-3. Failure surfaces for all sections analyzed are included in Appendix A.

"Bench" Option

Computed factors of safety range from 1.4 to 1.3 for Case 1 and Case 2, respectively. These results are based on the conservative assumption that the new fill along the inside perimeter of the embankment would remain fully saturated after the occurrence of sudden drawdown; the results are summarized in Tables 4-5 and 4-6. Failure sections for all sections analyzed are included in Appendix A.

4.2.4 Psuedo-Static Analyses

The pseudo-static analyses were performed to estimate the yield accelerations (K_y) to be used in the seismic risk analysis (see Seismic Analysis Report). Yield accelerations were determined assuming non-liquefaction and liquefaction in the upper sand layer. The K_y values for the upper sand liquefying are significantly lower than non-liquefaction because they are based on the consideration that the entire loose sand layer across the section has liquefied. This is a conservative assumption, as excess pore pressure less than 100 percent could be generated in more or less extended areas of the liquefaction-susceptible layer, depending on the amplitudes and duration of the shaking.

"Rock Berm" Option

Yield accelerations for portions of the islands where the upper portion of the sand layer does not liquefy range from 0.14 to 0.27 for Case 1 and from 0.09 to 0.12 for Case 2. Yield accelerations where the upper sand does not liquefy are only slightly sensitive to water levels in the reservoirs and slough. Where liquefaction occurs in the sand, yield accelerations are more sensitive to water levels in the reservoir and slough. The yield accelerations range from 0.03 to 0.12 for Case 1 and from 0.04 to 0.07 for Case 2. The results are summarized in Tables 4-2 and 4-3.

One analysis was performed to evaluate the effect of liquefaction of loose sandy fill in the reservoir side of the existing levee using Case 2. Yield accelerations towards the reservoir were found to slightly decrease for the case analyzed.

Failure surfaces for all sections analyzed are included in Appendix A.

"Bench" Option

Yield accelerations for the "bench" option are sensitive to water levels in the reservoir and slough for all cases analyzed. Yield accelerations for portions of the islands where the upper portion of the sand layer does not liquefy range from 0.1 to 0.14 for Case 1 and from 0.06 to 0.09 for Case 2. Where liquefaction occurs in the sand, yield accelerations range from 0.03 to 0.07 for Case 1 and from 0.01 to 0.08 for Case 2. The results are summarized in Tables 4-5 and 4-6.

The effect of liquefaction of loose sandy fill in the reservoir side of the existing levee was evaluated using Case 2. Yield accelerations towards the reservoir were found to decrease from 0.03 to 0.02 for the case analyzed.

Failure surfaces for all sections analyzed are included in Appendix A.

4.2.5 Post-Liquefaction Stability Analysis

Post-liquefaction stability analyses considered both circular and non-circular failure surfaces passing through the "liquefied layer" (assigned the post-liquefaction residual undrained strength). The development of earthquake-induced excess pore pressures in the existing levee materials was not considered, which is potentially unconservative. However, the entire loose sand layer was assumed liquefied, which is conservative.

"Rock Berm" Option

Computed factors of safety range from 1.9 to 3.0 for Case 1 and Case 2, respectively for those portions of the island where liquefaction of the upper portion of the sand layer does not occur. Where liquefaction of the upper portion of the sand layer occurs the computed factors of safety range from 1.3 to 2.4. The results are summarized in Tables 4-2 and 4-3. Failure surfaces for all sections analyzed are included in Appendix A.

"Bench" Option

Computed factors of safety range from 1.6 to 2.8 for Case 1 and from 1.5 to 2.1 for Case 2, for those portions of the island where liquefaction of the upper portion of the sand layer does not occur. Where liquefaction of the upper portion of the sand layer occurs the computed factors of safety range from 1.2 to 1.9 for Case 1 and from 1.1 to 1.7 for Case 2. For Case 2, a 2-foot layer of horizontal free draining fill was placed on the reservoir-side slope toe to increase the factor of safety from 1.0 to 1.1 for reservoir empty and high tide conditions. The results are summarized in Tables 4-5 and 4-6. Failure sections for all sections analyzed are included in Appendix A.

4.3 SUMMARY OF FINDINGS

Stability criteria can be met for embankments having either the “rock berm” or “bench” configurations as slough-side slopes. For both configurations, where the base of peat is deep, minor modification to the sections are required in order to meet all stability criteria. Specifically, the “rock berm” option requires a free draining horizontal stability berm at the reservoir-side slope toe to meet criteria for long term conditions for base of peat at elevations of –30 feet or lower. The “bench” option requires a free draining horizontal stability berm at the reservoir-side slope toe to meet post seismic stability criteria for those portions of the perimeter of the islands where liquefaction of the upper sand layer occurs and the base of peat elevation is -40 feet. Additional analysis should be performed as part of design to determine the final embankment configuration for the full range of existing levee geometry and subsurface conditions at the islands. End-of-construction stability analysis indicates that the embankments will require staged construction with the first stage being limited to a height of between 8 to 10 feet. Successive stages could be placed after eighty percent consolidation has occurred. This would occur after three months and 18 months, for peat with base elevations of –20 feet and –40 feet, respectively.

Based on the stability analysis presented in this section, the “rock berm” option appears to provide several advantages over the “bench” option as follows:

- Factors of safety for long term conditions toward the slough are higher, 2.0 to 1.8 compared with 1.6 to 1.5, suggesting less probability of an outward breach.
- Factors of safety for long term conditions toward the reservoir are higher, 1.9 to 1.7 compared with 1.6 to 1.5, suggesting less probability of an inward breach.
- Factors of safety for sudden drawdown conditions within the reservoir are higher, 1.6 to 1.5 compared with 1.4 to 1.3.
- Yield accelerations (and factors of safety for post seismic conditions) are equal or greater for nearly all conditions analyzed suggesting less deformation during earthquake events.

Table 4-1 – Stability Analysis Results, End-of Construction, "Rock Berm" Option (Peat at El. –40 feet)

Staged Construction	Factor of Safety		Implied shift of free field/under levee boundary towards slough (feet)
	$Su_{free\ field}$ = 200 psf $Su_{under\ levee}$ = 450 psf	$Su_{under\ levee} = Su_{free\ field}$	
Single stage	1.7	0.9	90 feet
Stage to 12 feet above reservoir bottom	1.8	1.2	40 feet
Stage to 8 feet above reservoir bottom	2.0	1.4	25 feet

[1] Slough water level = 3.5 feet.

Table 4-2 – Stability Analysis Results "Rock Berm" Option[1] (Base of Peat at El. –20 feet)

Existing Slough Side Slope	Rock Berm Slope	Condition	Water Elevation		Side Slope Considered	F.S.	Ky
			Slough	Reservoir			
1.4H : 1V (worst)	none	long term	-1.0	4.0	Slough	0.9	--
	3H : 1V	long term	-1.0	4.0	Slough	2.4	--
2.6H : 1V (average)	none	long term	-1.0	4.0	Slough	1.1	--
	3H : 1V	long term	-1.0	4.0	Slough	2.0	--
		long term	7.0	empty	Reservoir	1.9	--
		sudden drwdn	6.0	4.0/empty	Reservoir	1.6	--
2.6H : 1V w/o liquifiable sand layer	3H : 1V	seismic	-1.0	4.0	Reservoir	2.8[2]	0.14
		seismic	-1.0	4.0	Slough	2.7[2]	0.25
		seismic	3.5	Empty	Reservoir	2.1[2]	0.14
		seismic	3.5	Empty	Slough	3.0[2]	0.27
2.6H : 1V w/ liquefiable sand layer	3H : 1V	seismic	-1.0	4.0	Reservoir	1.8[2]	0.07
		seismic	-1.0	4.0	Slough	1.6[2]	0.08
		seismic	3.5	Empty	Reservoir	1.3[2]	0.03
		seismic	3.5	Empty	Slough	2.0[2]	0.12
5H : 1V (best)[3]	none	long term	-1.0	4.0	Slough	1.4	--
	2' layer rock fill	long term	-1.0	4.0	Slough	1.8	--

[1] Slough bottom = -25 feet.
[2] post-seismic factor of safety
[3] Slough bottom = -20 feet

Table 4-3 – Stability Analysis Results, "Rock Berm" Option[1] (Base of Peat at El. –40 feet)

Existing Slough Side Slope	Rock Berm Slope	Condition	Water Elevation		Side Slope Considered	F.S.	Ky
			Slough	Reservoir			
2.35H : 1V (worst)	none	long term	-1.0	4.0	Slough	1.1	--
	3H : 1V	long term	-1.0	4.0	Slough	1.8	--
2.6H : 1V (average)	none	long term	-1.0	4.0	Slough	1.2	--
	3.5H : 1V	long term	-1.0	4.0	Slough	1.8	--
		long term	7.0	empty	Reservoir	1.2	--
		long term	7.0	empty	Reservoir [2]	1.7	--
		sudden drwdn	6.0	4.0/empty	Reservoir [2]	1.5	--
2.6H : 1V w/o liquifiable sand layer	3.5H : 1V	seismic	-1.0	4.0	Reservoir [2]	2.6 [3]	0.09
		seismic	-1.0	4.0	Slough	1.9 [3]	0.11
		seismic	3.5	Empty	Reservoir [2]	2.0 [3]	0.09
		seismic	3.5	Empty	Slough	2.3 [3]	0.12
2.6H : 1V w/ liquefiable sand layer	3.5H : 1V	seismic	-1.0	4.0	Reservoir [2]	2.4 [3]	0.06
		seismic	-1.0	4.0	Slough	1.4 [3]	0.04
		seismic	3.5	Empty	Reservoir [2]	1.4 [3]	0.04
		seismic	3.5	Empty	Slough	1.8 [3]	0.07
3.5H : 1V (best)	none	long term	-1.0	4.0	Slough	1.3	--
	4H : 1V	long term	-1.0	4.0	Slough	1.6	--

[1] slough bottom = -30 feet.
[2] with u/s 2 foot thick horizontal rock berm
[3] post-seismic factor of safety

Table 4-4 – Stability Analysis Results, "Bench" Option - Sensitivity to Bench Elevation and Width

Peat Condition	Bench Elevation	Bench Width (feet)	Factor of Safety	
			Crest	Bench
Peat at –20 feet	6.0	34.0	1.33	2.00
	0.0	27.0	2.60	1.36
	3.0	31.0	1.64	1.63
Peat at –40 feet	6.0	36.0	1.36	1.39
	2.0	36.0	1.99	1.33
	2.0	65.0	1.94	1.46
	3.0	60.0	1.68	1.48
	3.0	65.0	1.68	1.52

[1] Long-term Condition (towards slough)
[2] Average slough-side slope used.
[3] Reservoir at 4.0 feet
[4] Water surface in slough at –1.0 feet

Table 4-5 – Stability Analysis Results, "Bench" Option [1] (Base of Peat at El. –20 feet)

Condition	Water Elevation		Side Slope Considered	Levee Crest		Bench [2]	
	Slough	Reservoir		Ky	F.S	Ky	F.S.
long-term	7.0	empty	Reservoir	--	1.6	--	--
	-1.0	4.0	Slough	--	1.6	--	1.6
sudden drawdown	6.0	4.0/empty	Reservoir	--	1.4	--	--
seismic w/o liquifiable sand layer	-1.0	4.0	Reservoir	0.14	2.8	--	--
	-1.0	4.0	Slough	0.095	1.6	0.094	1.7
	1.5	-2.5	Reservoir	0.13	2.5	--	--
	1.5	-2.5	Slough	0.12	1.8	0.12	2.2
	3.5	empty	Reservoir	0.14	2.1	--	--
	3.5	empty	Slough	0.11	1.8	0.12	2.6
seismic w/liquifiable sand layer	-1.0	4.0	Reservoir	0.07	1.9	--	--
	-1.0	4.0	Slough	0.027	1.2	--	--
	1.5	-2.5	Reservoir	0.05	1.5	--	--
	1.5	-2.5	Slough	0.053	1.4	--	--
	3.5	empty	Reservoir	0.027	1.2	--	--
	3.5	empty	Slough	0.063	1.5	--	--

[1] Average Slough-Side Slope used.
[2] Bench Elevation = 3.0 feet

Table 4-6 – Stability Analysis Results, "Bench" Option [1] (Base of Peat at El. –40 feet)

Condition	Water Elevation		Side Slope Considered	Levee Crest		Bench [2]	
	Slough	Reservoir		Ky	F.S	Ky	F.S.
Long-term	7.0	empty	Reservoir	--	1.5	--	--
	-1.0	4.0	Slough	--	1.5	--	1.7
sudden drawdown	6.0	4.0/empty	Reservoir	--	1.3	--	--
seismic w/o liquifiable sand layer	-1.0	4.0	Reservoir	0.094	2.1	--	--
	-1.0	4.0	Slough	0.06	1.5	0.08	2.0
	1.5	-2.5	Reservoir	0.086	1.9	--	--
	1.5	-2.5	Slough	0.085	1.7	0.125	2.5
	3.5	empty	Reservoir	0.07	1.5	--	--
	3.5	empty	Slough	0.078	1.7	0.110	2.7
seismic w/liquifiable sand layer	-1.0	4.0	Reservoir	0.058	1.6	--	--
	-1.0	4.0	Slough	0.009	1.1	--	--
	1.5	-2.5	Reservoir	0.075	1.7	--	--
	1.5	-2.5	Slough	0.029	1.2	--	--
	3.5	empty	Reservoir	0.015	1.1	--	--
	3.5	empty	Slough	0.03	1.3	--	--

[1] Average Slough-Side Slope used.
[2] Bench Elevation = 3.0 feet

5.1 GENERAL

The probability of embankment failure during normal operations is the aggregate of the probability of failure of identifiable failure modes. These failure modes include: 1) internal erosion and piping due to high exit gradient caused by excessive seepage, 2) erosion through cracks in the levees (existing) and embankment (engineered) caused by differential settlement or unstable slopes, and 3) overtopping caused by slumping or loss of freeboard due to slope failure or excessive settlement.

Houston and Duncan, (1978) predicted the aggregate annual probability of failure of the existing levees (0.02 for Bacon Island and 0.05 for Webb Tract) based on 27 years of historical observation of levee failures in the Delta. The predicted probabilities were calculated for 40 years of continued use of the islands for farming where the island elevations would continue to subside at a rate of 3 inches per year.

The engineered embankments will be much improved compared to the existing levees because the long-term factor of safety for stability meets the adopted design criteria of 1.5 or higher and seepage exit gradients will be 0.3 or lower. Based on the improvement of the engineered embankments over the existing levees, it is judged that the annual probability of failure would be approximately 100 times smaller than for the existing Bacon Island levees. Because the new embankments for both islands would be designed to meet the same criteria, the annual probability of failure was assumed to be the same. For this study, the annual probability of failure for the new embankments was estimated to be 2×10^{-4}.

The contribution to the annual risk of failure from the different failure modes is described in the following paragraphs. Appendix B provides further discussion on calculation of probabilities of failure.

5.2 PROBABILITY OF FAILURE DUE TO INTERNAL EROSION

Failure from internal erosion can occur due to high exit gradients caused by excessive seepage or through cracks in the existing levee or new embankment that may form during consolidation of the underlying soft soils. For the purposes of this evaluation, it has been assumed that protection against internal erosion due to cracking consisting of filter fabric between the existing levee and new embankment would be installed at selected locations around the islands where there is a greater likelihood of cracking to occur during construction. The filter fabric would provide piping protection for materials that are up-gradient of the fabric. Alternatives for mitigation measures against internal erosion failures due to cracking and piping are discussed in Appendix C.

The probability of failure from internal erosion due to high exit gradient caused by excessive seepage or cracking during normal operations was calculated using the method described in USBR (1997). The method requires the identification of steps leading to failure, assigning a probability of the occurrence of those steps, and multiplying the probability of occurrence of each of those steps to obtain the total probability of failure. The steps identified and the probability of each of the steps occurring are outlined in Appendix B. The calculated annual probability of failure due to internal erosion during normal operations (not including flood events) is 1.27×10^{-4}(considering weighted contribution for inward and outward flows).

5.3 PROBABILITY OF SEEPAGE FAILURE DURING FLOOD EVENTS

During high flood stage, a greater head difference between the water surface in the reservoir and the adjacent slough can exist compared to normal operations. Exit gradients at the toe of the new embankments during high flood stage (up to 300 year event) were calculated to be 25 percent higher (0.25 compared with 0.20) than during normal operations. These gradients are still less than those that could cause sand boils or piping. To estimate the probability of failure due to internal erosion during high flood stage, the contribution of different flood stages was proportioned to the percent change in the corresponding exit gradient as shown in Appendix B. On an annualized basis, the probability of failure due to internal erosion during flooding events is estimated to be 0.27×10^{-4}. The combined annual probability of failure due to seepage-induced piping under all tide and flood stages below elevation +10 feet is estimated to be $(1.27+0.27) \times 10^{-4}$ or 1.54×10^{-4}.

5.4 PROBABILITY OF OVERTOPPING CAUSED BY SLOPE FAILURE OR EXCESSIVE SETTLEMENT

The probability of failure due to overtopping caused by slumping or loss of freeboard due to slope failure or excessive settlement can be calculated as the difference between the annualized aggregated probability of failure and the probabilities of failure due to internal erosion during normal operations included high flood events. This calculated probability of failure is estimated to be $(2 - 1.54) \times 10^{-4} = 0.46 \times 10^{-4}$. This calculated probability of embankment failure should be relatively lower due to the following:

- The embankments are designed for a long term factor of safety of 1.5 and higher.
- Foundation soil strengths and embankment strengths are based on back-calculated strengths from a failure on Webb Tract and likely represent some of the lower strengths for the islands.
- There will be opportunity to assess settlement and stability during the five-year construction period. Areas of the islands that exhibit settlement or stability problems could be addressed during construction.

6.1 SEEPAGE

The findings from the seepage analysis were based on two representative sections for Webb Tract and one section for Bacon Island. The cross sections at Webb Tract island were selected for the “narrowest” and “widest” slough width across reservoir island and neighboring island. The section across Bacon Island represents a case that lies in-between the “narrowest” and “widest” cases of Webb Tract. These cross sections represent somewhat a bounding of the seepage conditions. The following major findings emerged from the seepage evaluations.

- Seepage mitigation measures should be considered to control undesirable seepage flooding effects on adjacent islands that may occur as a result of the reservoirs.
- Seepage control by interceptor wells placed on the levees of the reservoir islands, as proposed, appears effective to control undesirable seepage effects. Well spacings of a minimum of 160 feet would be required where the adjacent slough is the narrowest. Wider well spacings could be used at other locations. The required pumping rates of about 6 to 8 gpm appear to be reasonable and manageable.
- Success of an interceptor well system will be a function of proper design, construction, maintenance, and monitoring.
- Other seepage control alternatives should be further investigated because of their potential engineering merits.

Based on the results of the current study, the following recommendations are made:

- Sensitivity analysis reported in URS (2000) demonstrated that increases in the permeability of the sand layer significantly increase calculated seepage volumes. Site specific pump tests located at potential seepage area on Webb Tract and Bacon Islands are recommended for design of the interceptor system.
- Pilot test borings should be drilled along those portions of Bacon Island and Webb Tract where interceptor wells are planned. Data gathered from the borings should be used for final design of the well system.
- During final design, Webb Tract and Bacon Islands should be surveyed for potential seepage problem areas. Potential seepage areas should be analyzed individually using parameters obtained from pump tests and additional borings.
- Test interceptor well sections should be installed and tested based on data collected from pump tests and pilot borings. Results of the test sections should be incorporated into the final design.

6.2 EMBANKMENT CONFIGURATION

Two configurations for the project’s embankments have been evaluated by extensive stability analyses of two sections selected to be representative of the lowest and highest elevations at which the base of the underlying peat layer is found in the two islands. Stability analyses were performed for the more severe situations expected at the reservoir islands. The calculated factors of safety have been compared to the project’s stability criteria, and judgments were made of the adequacy of the proposed project in regard to embankment stability. The resulting conclusions and recommendations are:

- Construction of the levee strengthening fills must be implemented in a manner to prevent stability failures due to the new fill loads. This will require carefully planned staged construction, and monitoring to observe the behaviors as the fill is placed. The staged construction will require a construction period estimated to extend over 4 to 6 years.
- Both the "rock berm" and "bench "option" can be constructed to meet the project's required stability criteria. For some combinations of existing reservoir bottom elevation and base of peat elevation reservoir-side slope free draining toe berms are required to meet stability criteria.
- Based on the stability analysis presented in this section, the "rock berm" option appears to provide several advantages over the "bench" option as follows:
  - Calculated factors of safety for all analysis cases are greater than calculated for the "bench" option suggesting a lower probability of failure during normal operations.
  - Calculated yield accelerations are generally greater than for the "bench" option suggesting less earthquake induced deformation. Deformations are addressed in the Seismic Analyses Report.
  - Fill volumes for new embankments are significantly less due to less consolidation deformation under new embankment and the absence of setback.
- A probability of failure of the embankments during normal operations based on engineering judgement was presented.

Based on the results of the current study, the following recommendations are made:

- Implement an extensive subsurface exploration program along the reservoir island levees, followed by stability evaluations and site-specific detailed design and construction to provide adequate embankment stability during design. These steps will be essential to achieve safety and effectiveness of the proposed embankment system.
- Conduct of the subsurface exploration program should include sample collection and laboratory testing designed to evaluate the potential for liquefaction of the reservoir side of the existing levees, the variation of the strength of peat under levee and free field peat and the transition between them, and the change in strength in the peat as it consolidates under the new embankment.
- Conduct a survey of Webb Tract and Bacon Island to determine the extent and thickness of existing rockfill on the slough-side slopes. Where rockfill exists on the slough-side slopes, rock berm slopes required to meet stability criteria may be reduced.
- Implement a test fill section during design for the preferred embankment geometry at locations where the base of peat is located at elevations –20 feet and –40 feet. The test fill program would provide valuable information regarding consolidation rates and ultimate settlement for estimating the time required for staged construction. The test fills should be monitored using piezometers, settlement survey monuments, and visual observation during and after construction.

- Include in the final design a filter fabric between the new embankment and existing levee to provide piping protection for materials that are up-gradient of the fabric. Determination of the locations along the reservoir embankments for filter fabric as a piping mitigation measure should be made during future engineering studies.

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